CHAPTER E-6 SEISMIC PIER FAILURE

E-6.1 Key Concepts

E-6.1.1 Description of Potential Failure Mode

This potential failure mode relates to gated spillways with intermediate piers. Concrete piers separate the spillway crest into bays, with each bay regulated by a spillway gate. End piers of spillway crest structures that support soil at the outside face are considered walls and should be evaluated according to Chapter E-7. Large inertial loads can be generated for spillway piers during seismic loading. Spillway gates are typically anchored to the spillway piers, which imposes additional loads into the piers during an earthquake, as static and hydrodynamic loads from the reservoir are transferred from the spillway gates into the piers. Water can be stored against the piers upstream of the gates, creating additional hydrostatic and hydrodynamic loading on the piers. Pier loading in both the cross-canyon direction and the upstream/downstream direction are important to consider for this potential failure mode. Catastrophic failure of the spillway piers can directly lead to gate failure if the gates lose their support or if excessive pier deflections load the gates laterally and cause them to buckle. Figure E-6-1 shows how high tensile stresses can be generated near the base of the piers due to cross-canyon loading and bending.
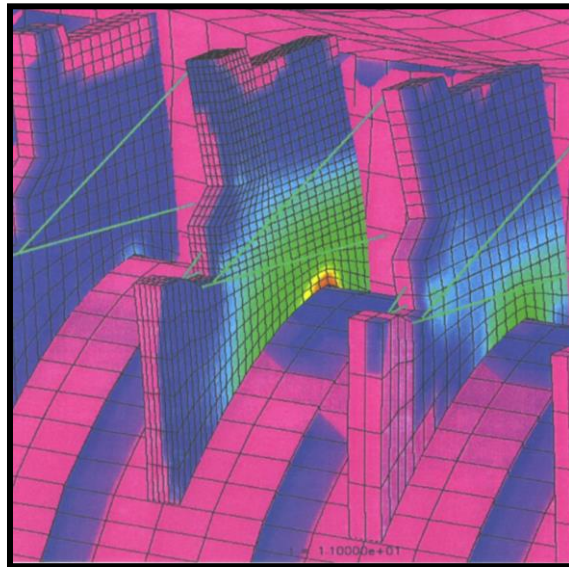

Figure E-6-1 – Stress Plot from Pier Finite Element Analysis

E-6.1.2　　　　　Reservoir Water Surface Elevation

The reservoir water level on the spillway piers and gates is a key parameter since it affects the loading on the piers both statically and dynamically and also affects the consequences of a pier failure due to the effect on the breach outflow.

E-6.1.3　　　　　Pier Geometry

The pier geometry (height and thickness of the pier) will affect how the pier is excited during an earthquake. The lateral support at the top of the piers is also critical.

E-6.1.4　　　　　Concrete Tensile Strength

Before a pier can fail due to moment capacity exceedance, the tensile strength of the concrete at the face of the pier must be exceeded. See the section on Concrete Gravity Structures for a discussion of concrete tensile strength and the section on Reinforced Concrete Failure Mechanisms for evaluation of the section cracking moment.

E-6.1.5　　　　　Moment Capacity

The moment capacity of the pier will be a function of the steel reinforcement provided at a given location within the spillway pier.

E-6.1.6　　　　　Shear Capacity

The shear capacity of a pier will, in most cases, consist of the shear capacity of the concrete. For cases where steel shear reinforcement is provided, the shear capacity will be a combination of the reinforcement shear capacity and the concrete shear capacity. If the concrete cracks due to large tensile stresses during an earthquake, the shear capacity may be reduced. However, reinforcement crossing cracks will become stressed due to dilation of the cracks during shearing, thus increasing the normal stress and frictional shear strength across the cracks.

E-6.1.7　　　　　Seismic Hazard

Most spillway piers will have some reserve capacity beyond the stress levels created by full reservoir static loads. However, the level of seismic loading in combination with the reservoir level at the time of loading will determine whether the piers are overstressed and, if so, to what level.

E-6.1.8   Number of Piers

Multiple spillway piers on a given project will typically increase the probability of a pier failure with the outcome varying from a single pier failing to all the piers failing. Multiple spillway pier failures also create the potential for a larger breach outflow and higher potential life loss. A single pier failure has the potential to result in failure of the two adjacent spillway gates.

E-6.1.9   Gate Loads

The most common type of spillway gate at Reclamation and Corps dams is the radial gate. Radial gates transfer the loads from the semi-circular gate leaf thorough radial gate arms to a trunnion, which is often attached to or embedded in a spillway pier. The loads on the gate are magnified during an earthquake when hydrodynamic loads are transferred to the trunnion in addition to the normal hydrostatic loading. The loads transferred to the pier via the trunnion will be a function of the gate size, the reservoir level at the time of the earthquake and the seismic loads. For other types of spillway gates (drum gates, large vertical lift gates, Obermayer gates, etc.), the gates may or may not contribute loads to the spillway piers. Even if the gates are not designed to concentrate loads on the spillway piers, the inertial effects of the spillway piers and gates during an earthquake could be significant.

E-6.1.10   Trunnion Anchorage for Spillway Radial Gates

For radial gates, trunnion anchorage is typically provided in spillway piers to anchor the trunnion pin and transfer the gate loads in to the pier. During an earthquake, the anchorage may be stressed to a level well beyond its design capacity and, as a result, the anchorage should be evaluated for seismic loads. Traditionally, spillway radial gates are analyzed by pseudo-static methods. The combined static and hydrodynamic loads from the gate are transferred to the trunnion pin and then to the trunnion anchorage, placing the anchorage in tension. When the peak or near-peak gate loads from an earthquake are applied to the trunnion anchorage, an initial evaluation may indicate that the ultimate capacity of the anchorage will be exceeded. However, a time-history analysis may indicate that the anchorage will start to strain but the load will not be sustained to the point that the anchorage will fail. This may be the case in certain instances especially if the trunnion anchorage is not bonded to the pier concrete. For some anchorage details cork was used to isolate the steel plates forming the anchors. In this case, the anchor may

elongate before failure would occur. If the anchors are fully bonded, the only portion of the anchor that will be free to elongate before bond is broken between the anchor and the concrete is the short section of bar between the concrete pier and the nut securing the end of the anchor. In this case the strain required to fail the anchor will be relatively small. However, some disbonding may occur between the anchor and the concrete resulting in additional length that can strain before failure. For other types of spillway gates, the connections that attach the gates to the spillway structure (such as hinges for drum gates, or gate slots for fixed wheel gates) should be evaluated for structural integrity when subjected to earthquake loads.

E-6.1.11   Spillway Bridges

Spillway bridges are typically provided across gated crest structures. The bridges provide two functions – 1) access across the crest structure and 2) support for the hoist equipment used to operate the gates. The access and hoist bridges may be two separate structures or may be combined into one bridge. In some cases, bridge decks sit in recessed block-outs at the tops of the piers, thereby limiting the amount of cross-canyon deflection that can occur before the strength of the bridge deck is mobilized. In other cases, a bridge deck may sit on top of the piers with bearing connections, such that the bridge deck offers no significant lateral support. An example of a spillway bridge spanning a gated spillway crest structure is shown on Figure E-6-2.

Figure E-6-2 – Example of a Spillway Bridge Spanning a Gated Spillway Crest Structure

Bridges may affect the loading and the structural capacity of spillway piers during seismic loading. Bridges will generally add a compressive load to the piers that may be partially offset with vertical upward seismic accelerations. Depending on how the bridge is attached to the piers and the end walls, the bridge may be accelerated with the piers and transmit inertial loads to the top of the piers without providing much, if any, lateral support or the bridge may serve as a strut to create a structural frame across the crest structure, reducing the moment and shears at the base of the pier. The geometry and interaction of the piers and spillway bridges should be considered in the pier analysis. If there is uncertainty as to how the bridge will affect pier performance during an earthquake, or uncertainty as to whether the bridge will survive a given seismic event, separate analyses can be performed with and without the bridge in place. If there is the potential for the bridge itself to fail during an earthquake, the bridge should be analyzed and the potential impact loading from the failed bridge on the spillway gates should be evaluated.

E-6.1.12   Screening

This potential failure mode only applies to spillways that are gated. If the spillway has piers but no gates (an unlikely situation), there would not be an uncontrolled release as a result of pier failure. If the reservoir level is typically below the spillway crest elevation and only infrequently results in water on the gates, it may be possible to build the case that the probability of failure for this potential failure mode is remote. Finally, if a simple pseudo-static analysis indicates moment and shear stresses that are within the capacity of the pier and there are no other issues with the trunnion anchorage or deflections of the pier, this potential failure mode can be classified as being remote.

E-6.2 Structural Analysis

Simplified methods for estimating the moments and shears in a member during seismic loading for this potential failure mode can be found in the section on Reinforced Concrete Failure Mechanisms. If these simplified approaches indicate that the computed moments and shears are less than the expected moment and shear capacities, the results will generally be conclusive. If the results indicate overstressing, more refined analyses will be beneficial to fully evaluate this potential failure mode. Specifically, a non-linear time history finite element analysis will provide a more complete picture of the extent of overstressing that will occur, including the

magnitude of the overstress and the number of excursions when the capacity of the structure is exceeded. The non-linear finite element analysis will provide the risk analysis team with a much clearer picture of the expected damage resulting from an earthquake.

The pier analysis should evaluate the pier in both the cross-canyon and the upstream-downstream directions. Directionality of the ground motions is important. If directivity is included in the development of the ground motions, it would not be appropriate to arbitrarily apply the largest component in either the upstream-downstream or cross-canyon direction. If directivity is not included, it is appropriate to vary the application of the two horizontal components to determine the critical condition. Inertial loads on the piers will be more critical in the cross-canyon or weak axis direction of the pier. While it is generally more critical to consider the reservoir loading in the upstream-downstream direction due to the large loading that is transferred from the spillway gates to the piers, the cross-canyon reservoir loading may also be important to consider, especially if the reservoir loads the piers upstream of the gates to a significant extent. During seismic loading, there may be a differential loading of the pier from one side to the other and this loading at the upstream portion of the pier may torque the pier and create localized high stresses. For pier analyses, considering both axial loads and flexural loads using a P-M interaction diagram as presented in the section on Reinforced Concrete Failure Mechanisms is important in evaluating this potential failure mode.

Hydrodynamic interaction must be included in determining gate loads to be applied to the piers. This is typically done using Westergaard's added mass, which is a function of the seismic coefficient related to the peak horizontal accelerations and applied to the gate as a load, as described in the section on Seismic Failure of Radial Gates. An alternative to using Westergaard's added mass is to create a three-dimensional finite element model, in which multiple piers, gates and fluid elements to simulate the reservoir are modeled as shown on Figure E-6-3 and perform a time-history analysis. If this approach is taken, significant verification and testing of the model is essential to ensure the reservoir is responding and interacting with the dam and spillway crest structure correctly without separating from the structures or reducing load when the structural elements deform. While the modeling and analysis are more complicated, time-consuming, and expensive, it can provide additional insights regarding pier

performance during an earthquake in critical cases where performance using added mass and pseudo-static analysis creates a significant level of uncertainty in final risk estimates.

Depending on the level of analyses that are available for evaluating the potential for pier failure during an earthquake, the estimates for many nodes on the event tree may, by necessity, be subjective (see the section on Subjective Probabilities).
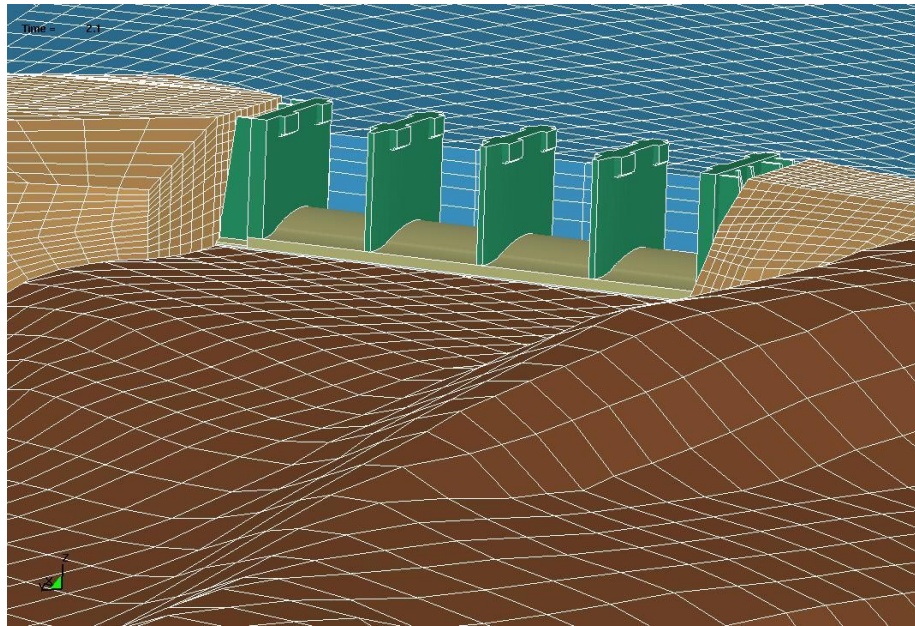

Figure E-6-3 – Example of a Non-linear Finite Element Model Including the Reservoir Water.

E-6.3 Event Tree

Figure E-6-4 is an example of an event tree for this potential failure mode. The event tree consists of a number of events that lead from initiation, through progression, to breach of the reservoir through gates that fail as a result of a pier failure. The first node of the event tree represents the starting reservoir water surface elevation and the second node represents seismic load ranges. The combination of these first two nodes represents the combined load probability and affects the loading on the piers in both the upstream-downstream and cross-canyon direction.

The remaining nodes in the event tree represent the conditional probability of failure given the load probability. They are patterned after the general event tree for failure of reinforced concrete

members, as described in the section on Reinforced Concrete Failure Mechanisms. First the likelihood of the concrete cracking from internal tensile stresses that develop as a result of applied seismic loads is evaluated. As part of this evaluation, the potential for crushing of the concrete is also checked. If the concrete is not expected to crack, the shear capacity of the intact pier is checked. If the shear capacity of the pier is exceeded, the potential kinematic instability of the pier resulting from inadequate shear capacity is evaluated to determine if the pier would slide or topple to the extent necessary to fail the adjacent spillway gates. If the concrete is expected to crack, then the likelihood of the reinforcement yielding is evaluated. The likelihood of the reinforcement yielding is typically estimated based on the computed moments at a critical section of the pier (typically the base of the pier) and the estimated moment capacity of the member at that section. The reinforcement could remain elastic or yield, depending on the resulting demand-to-capacity ratio as discussed in the section on Reinforced Concrete Failure Mechanisms. If the reinforcement remains elastic, then the likelihood that the shear capacity is exceeded is checked assuming a cracked concrete section including the effects from clamping forces of the reinforcement, if appropriate. If the shear capacity is exceeded, the likelihood of kinematic instability of the pier under the loading is estimated. If the reinforcement yields, the likelihood for shear failure is estimated for a cracked concrete section with yielded reinforcement, followed by an evaluation of kinematic stability. If the shear capacity is not exceeded, then the likelihood of uncontrolled non-linear pier displacement leading to gate collapse is estimated. Additional details regarding key factors and considerations for each one of these nodes is provided in the section for Reinforced Concrete Failure Mechanisms.
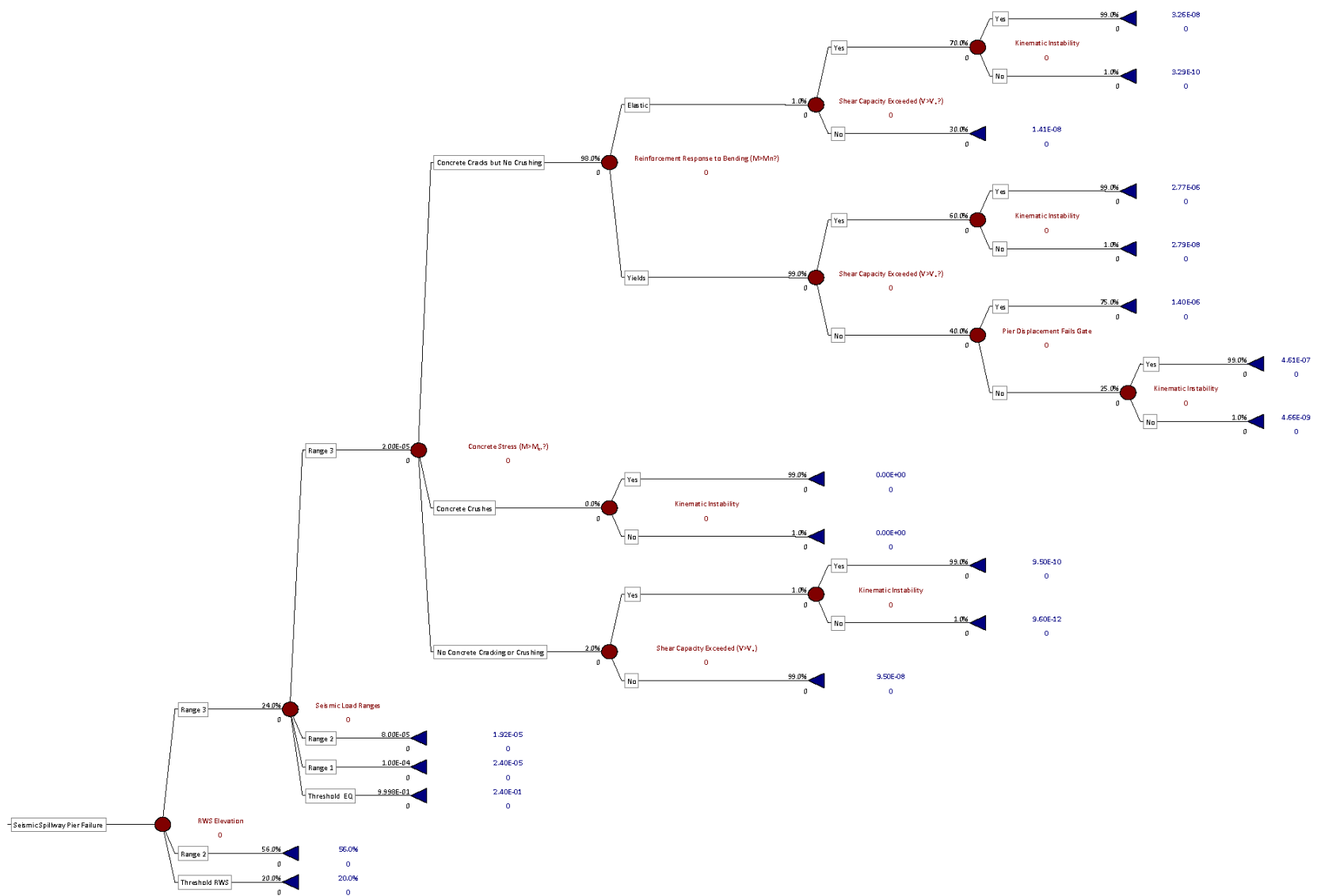
Figure E-6-4 – Example Event Tree for Seismic Spillway Pier Failure

H-2-2

E-6.3.1   Event 1 – Reservoir Water Surface Elevation

Reservoir load ranges are typically chosen to represent a reasonable breakdown of the entire reservoir range from the normal water surface (i.e., maximum controllable reservoir water surface elevation that may be associated with the top of active conservation storage or the top of joint use storage) and an elevation in the lower half of the pier in which stresses in the piers from this loading would not be expected to lead to failure associated with this potential failure mode. The number of load ranges depends on the variation in failure probability, and should be chosen, to the extent possible, to avoid large differences in failure probability at the top and bottom of the selected ranges. Historical reservoir elevation data can be used to generate the probability of the reservoir being within the chosen reservoir ranges, as described in the section on Reservoir (and River Stage) Exceedance Probabilities.

E-6.3.2   Event 2 – Seismic Load Ranges

Seismic load ranges are typically chosen to provide a reasonable breakdown of the earthquake loads, again taking into account the variation in failure probability to avoid large differences between the top and bottom of each range. The total range should include loading from the threshold level at the lower end where the pier is expected to perform satisfactorily, to the level at which failure is nearly certain at the upper end. The total load range is generally subjective depending on the available seismic hazard curves and analysis information. As an example, the lower end of the total range may be determined based on the level of seismic loading at which code design values are just exceeded. Seismic hazard curves are used to generate the probability distributions for the seismic load ranges as described in the sections on Probabilistic Seismic Hazard Analysis.

E-6.3.3   Event 3 – Concrete Stress

This node evaluates whether or not the tensile capacity of the concrete will be exceeded at the outer face of the pier and whether tensile cracking of the concrete will initiate. This can be determined by comparing calculated applied moments for the pier to the cracking moment of the section under investigation. If the cracking moment is exceeded, cracking will occur. The following equation can be used to calculate the cracking moment:

$$Mcr = fr\ Ig\ /yt \qquad\qquad \textbf{Equation E-6-1}$$

f_r = concrete tensile strength

I_g = moment of inertia of the gross concrete section

y_t = distance from the centroidal axis of gross concrete section to the extreme tension fiber

Another approach is to compare tensile stresses in the pier to the tensile strength of the pier concrete. If test data are available, this information should be used to determine the tensile strength of the pier concrete. In the absence of actual data, the tensile strength of concrete can be determined using the considerations discussed in the section on Concrete Gravity Dams. If a finite element analysis has been performed, the tensile stresses at the faces of the pier can be compared directly to an estimated tensile strength value to evaluate this node. If a time history analysis is performed, the number of excursions in which the tensile stresses exceed the tensile strength of the concrete can be determined. If many excursions occur it is very likely the pier section will crack through its entire thickness. If one or two isolated excursions occur, a less likely estimate for this node will be justified. The lateral extent over which cracking is predicted to occur is also important in terms of evaluating the remaining nodes of the event tree.

There is the potential for large compressive stresses to cause crushing of the pier concrete and this is checked as part of this node. Most large spillway structures, including the piers, are typically massive and under reinforced, so concrete crushing is not a common issue.

E-6.3.4   Event 4 – Reinforcement Response to Bending
If the cracking moment is exceeded, this node evaluates the likelihood of the reinforcement yielding for each earthquake load range. This node is evaluated by comparing the moments calculated from the earthquake (including both upstream-downstream and cross-canyon loading) to the expected moment capacity of the pier when the reinforcing steel reaches its yield value. If a time history analysis is performed, the moment demand-capacity ratio can be calculated at time steps during an earthquake. Axial tension resulting from vertical components of the earthquake or from vertical reaction components from the radial gates should also be considered. The number of excursions when the demand-capacity ratio exceeds 1.0 and the extent to which it is

exceeded are key factors for estimating this node of the event tree. A flexural yielding section response curve for estimating yielding probability of a reinforced concrete member is provided in the section on Reinforced Concrete Failure Mechanisms.

If moments during the earthquake are less than the yield moment capacity of the pier, then the section under investigation is generally considered to remain elastic during the earthquake.

E-6.3.5 Event 5 – Shear Capacity Exceeded

This node includes evaluation of the shear capacity of the pier in the context of three potential member conditions based on the outcomes of preceding nodes. Specifically, at this node the condition of the pier is one of the three options below:

- Uncracked and has demonstrated the ability to carry applied moments based on the expected concrete tensile strength as if the section were unreinforced.
- Cracked but with reinforcement carrying applied moments and forces within the expected elastic (unyielded) capacity of the section.
- Cracked with reinforcement that has yielded resulting in the onset of non-linear behavior of the member.

Regardless of which of the three states the pier is in at this node the shear stresses during the earthquake are compared to the expected shear capacity of the pier based on its damaged state, if applicable. As an example, if a construction joint exists within the spillway pier that is not fully bonded; this will create a weak plane in the pier through which a shear failure can occur. In this case, the failure location and the shear capacity should be adjusted accordingly. Additionally, if the trunnion anchorage for the spillway gates is located above a critical construction joint (that is at least partially unbonded), the total load resisted by the anchorage will have to be carried across the construction joint. If the shear capacity is exceeded, failure of the spillway pier and subsequent failure of the spillway gates is possible. Similarly, if the reinforcement for the section under investigation has yielded, the likelihood that the reinforcement at that section would also provide some additional shear capacity in the form of shear friction reinforcement would be considered less likely than for an elastic section.

Regardless of the damaged condition of the section, if the shear capacity is exceeded, a brittle failure of the pier may occur, with little to no chance of intervention. A discussion regarding various methods for estimating shear capacity of reinforced concrete members and an example shear response curve for estimating probability of a shear failure of a reinforced concrete member for various demand capacity ratios is provided in the section on Reinforced Concrete Failure Mechanisms.

E-6.3.6 Event 6 – Pier Displacement Fails Gate

Even if a spillway pier does not fail catastrophically in shear, once the reinforcement yields, the pier may deflect enough to laterally load a spillway gate and cause the spillway gate to fail. For radial gates, a certain amount of deflection will be required to load the main horizontal structural elements of the gate. The gate seals and the outer edge of the skinplate provide some room for deflections to occur without impacting horizontal beams. As discussed in the section on Reinforced Concrete Failure Mechanisms, the extent to which the pier may deflect before uncontrolled non-linear displacement occurs can be evaluated using displacement criteria (Gulkan and Sozen, 1974; Otani and Sozen, 1974), which indicates that reinforced concrete structures can achieve 2 to 3 times the yield displacement of the structure and still be capable of supporting applied loads. If the calculated displacements exceed 2 to 3 times the yield displacement, the piers may topple or the gates would likely be subjected to the full weight of the piers.

E-6.3.7 Event 7 – Kinematic Instability

This node evaluates the kinematic instability of the damaged member. Specifically, at this node in the event tree the member under investigation is either inadequate to carry applied seismic loads in shear or has yielded to the extent necessary that significant non-linear displacements have occurred. If the shear capacity is exceeded the pier can become unstable as a result of earthquake loading due to either sliding or toppling depending on the geometry of the pier and the locations of the more severely damaged sections. If deflection criteria is exceed, the pier will likely topple during the earthquake, but could slide post-seismically under sustained (static) loads due to the reduced shear capacity of the damaged section. However, it is also possible that stability of the pier could be maintained such that the adjacent gates do not fail, particularly for

piers that are braced at the top. Additionally, in general, piers with a larger base to height ratio will be more stable than piers with a smaller base to height ratio.

E-6.4 Statistical Considerations for Multiple Piers

Spillways with multiple piers can have a variety of potential pier failure outcomes, ranging from one pier failing to all the piers failing. Pascal's triangle, shown in Figure E-6-5, provides the number of combinations of each outcome for a given number of piers. The section on Seismic Failure of Spillway Radial (Tainter) Gates contains a discussion and figure that provides Pascal's triangle coefficients.

Number of Gates																
0								1								
1							1		1							
2						1		2		1						
3					1		3		3		1					
4				1		4		6		4		1				
5			1		5		10		10		5		1			
6		1		6		15		20		15		6		1		
7	1		7		21		35		35		21		7		1	
8	1	8		28		56		70		56		28		8		1
9	1	9		36		84		126		126		84		36	9	1
10	1	10	45		120		210		252		210		120	45	10	1
11	1	11	55		165		330		462	462		330	165	55	11	1
12	1	12	66	220		495		792	924	792	495		220	66	12	1
13	1	13	78	286	715		1287	1716	1716	1287		715	286	78	13	1
14	1	14	91	364	1001	2002	3003	3432	3003	2002	1001	364	91	14		

Figure E-6-5 – Pascal's Triangle for Multiple Gate Failure Probability Coefficients

As shown on the highlighted yellow row of Figure E-6-5, for a spillway that has five spillway piers, the coefficients represent the number of combinations of each outcome, as follows:

> 0 piers failing – 1 combination
> 1 pier failing –   5 combinations
> 2 piers failing – 10 combinations
> 3 piers failing – 10 combinations
> 4 piers failing – 5 combinations
> 5 piers failing – 1 combinations

These numbers are used as coefficients in the probability equations. For example, Table E-6-1 provides the equations for various failure outcomes (from zero to five piers failing) based a spillway with five piers (see far left column). The total at the bottom is the probability of one or more piers failing (i.e. is the sum of the rows from 1 to 5 piers failing and does not include the

probability of zero piers failing). The general form of the Pascal's triangle equation for calculating the probability of various failure outcomes (an outcome being the number of piers that fail during a seismic event) is provided in the Seismic Failure of Spillway Radial (Tainter) Gates section.

It should be noted that this approach assumes that the failure probability of each pier is independent of the failure probabilities of other piers. This is not necessarily the case. It holds true if there is an unknown defect that is unique to each pier which controls its failure probability. On the other hand, if it were known that one pier was near failing (not necessarily related to a unique defect), then this would affect the failure probabilities for the other piers. However, in general, the Pascal's triangle approach seems reasonable, in that if the failure probability of a single pier is small, the failure probability of multiple piers is also small; whereas, if the probability of a single pier is high, the failure probability of multiple piers is also high, as illustrated in Table E-6-1.

Typically, the combination of lower seismic load and lower reservoir elevation will have a significantly greater likelihood than higher seismic load and higher reservoir elevation, for each identified load range. Therefore, assigning equal weight to the boundary failure probabilities for a load range is generally conservative. This is especially true when there is a large range of failure probabilities at the boundaries of the load range (in which case it may be appropriate to look at smaller load ranges). Thus, the event tree is often run using conditional failure probabilities that represent both the average of the ends of the ranges, and the lower ends of the ranges. If there is a large difference in the results, then additional refinement or weighting is probably needed (see also the section on Event Trees).

Table E-6-1 – Example Pascal's Triangle Failure Probability Estimates

Probability for Single Pier → Failure		0.001	0.05	0.16	0.94
No. of Piers Failing	Equation for "x" Piers Failing	Probability for "x" Piers Failing	Probability for "x" Piers Failing	Probability for "x" Piers Failing	Probability for "x" Piers Failing
0	$1P^0(1-P)^5$	0.995	0.774	0.418	7.8E-7
1	$5P^1(1-P)^4$	0.005	0.204	0.398	6.0E-05
2	$10P^2(1-P)^3$	1.0E-05	0.021	0.152	1.9E-03
3	$10P^3(1-P)^2$	1.0E-08	0.001	0.029	0.03
4	$5P^4(1-P)^1$	5.0E-12	3.0E-05	0.003	0.234
5	$1P^5(1-P)^0$	1.0E-15	3.0E-07	1.0E-04	0.734
Total Probability of One or More Piers Failing		0.005	0.226	0.582	1.00

E-6.5 Consequences

Consequences are a function of the number of piers that fail and the reservoir level at the time of failure (which will determine the breach outflow). For a typical spillway pier, failure of the pier will likely lead to the failure of two spillway gates, since a pier provides support for the gates on either side. There are two extremes for how pier failure will lead to gate failure, when multiple pier failure is considered. The worst case is that alternating piers fail, so that all the gates can be failed in the quickest manner (2n case).   In this case failure of each pier results in the failure of two gates (except for an odd number of piers when failure of the last pier only fails one gate). This can be visualized by looking at the gate sequence failure identified in Figure E-6-6. The

best case is that the piers fail in succession from one end of the spillway crest to the other (n+1 case). In this case, the first pier failure results in two gates failing, and subsequent pier failures result in one additional gate failing. This n+1 case can be visualized in the gate failure sequence shown in Figure E-6-7.
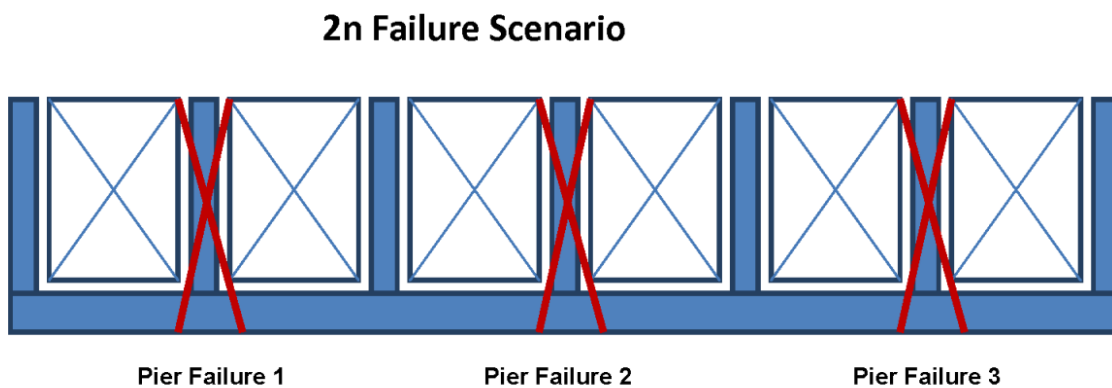
2n Failure Scenario

Figure E-6-6 – Pier Failure 2n Scenario Failing All Six Gates

n+1 Failure Scenario
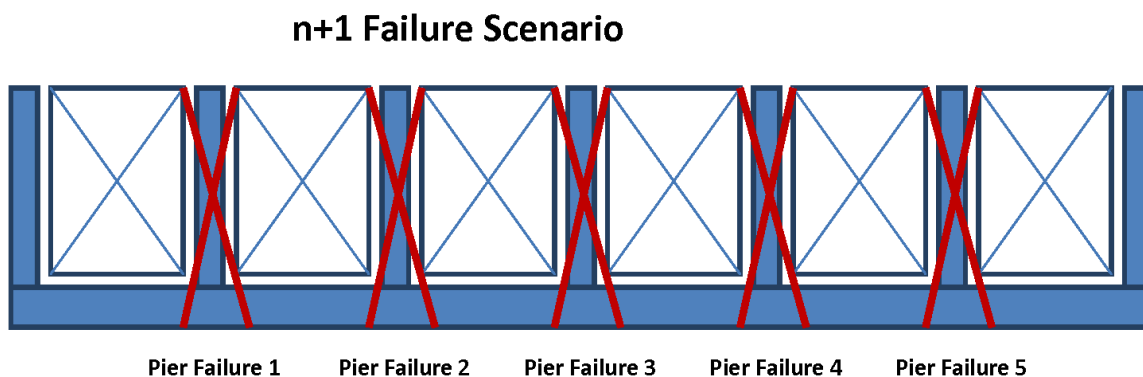
Figure E-6-7 – Pier Failure n+1 Scenario Failing All Six Gates

Tables E-6-2 and E-6-3 provide an example of how the consequences are affected for the two extreme cases. In this example at least 4 gates need to fail to exceed the safe channel capacity of 160,000 ft^3/s. However, smaller flows from fewer pier failures could impact recreationists adjacent to the river. Life loss can be estimated from these breach flows and from the estimated

population at risk that would be exposed to the breach outflows using the procedures outlined in the section on Consequences Evaluation. To estimate a weighted life loss for each seismic load and reservoir elevation range, the estimated life loss associated with various pier failure outcomes (i.e. number of piers that fail) is multiplied by the conditional failure probability for the corresponding outcomes. The total (sum) conditional life loss estimate is then divided by the total (sum) conditional failure probability estimate to arrive at the weighted average life loss value. Example calculations for weighted life loss are shown in Tables E-6-2 and E-6-3, for a given reservoir elevation and single pier failure probability. The main difference is that the life loss increases more rapidly with more pier failures using the 2n case, since more gates are failing as a result.

Table E-6-2 – Weighted Average Life Loss – Single Pier Failure Probability (P) = 0.16, RWS El 458, n+1

Number of Piers Failing	Probability of Failure Equations	Probability (P_x) of (x) Piers Failing	Expected Life Loss Value	Life Loss for (x) Piers Failing x (P_x)
1	$P_1 = 5(P)^1(1-P)^4$	0.398	16*	6.37
2	$P_2 = 10(P)^2(1-P)^3$	0.152	23*	3.50
3	$P_3 = 10(P)^3(1-P)^2$	0.029	30*	0.87
4	$P_4 = 5(P)^4(1-P)^1$	0.003	147	0.44
5	$P_5 = 1(P)^5(1-P)^0$	1.0E-04	164	0.02
Totals		0.58		11

*Loss of life due to recreational activity only

For the n+1 case, the Weighted Average Life Loss = 11/0.58 = 19. For the 2n case, the Weighted Average Life Loss = 16/0.58 = 28. The consequences for each seismic and reservoir load range are considered in the same way as the conditional failure probability. If the average of the load range boundaries produces risks that are considerably different than using the low value for the load range boundaries, additional refinement or weighting should be considered.

Table E-6-3 – Weighted Average Life Loss – Single Pier Failure Probability (P) = 0.16, RWS El 458, 2n

Number of Piers Failing	Probability of Failure Equations	Probability (P_x) of (x) Piers Failing	Expected Life Loss Value	Life Loss for (x) Piers Failing x (P_x)
1	$P_1 = 5(P)^1(1-P)^4$	0.398	16*	6.37
2	$P_2 = 10(P)^2(1-P)^3$	0.152	30*	4.56
3	$P_3 = 10(P)^3(1-P)^2$	0.029	164	4.76
4	$P_4 = 5(P)^4(1-P)^1$	0.003	164	0.49
5	$P_5 = 1(P)^5(1-P)^0$	1.0E-04	164	0.02
Totals		0.58		16

* Loss of life due to recreational activity only

E-6.6 Results

Due to the large number of load ranges, it is usually easier to enter the event tree as rows and columns in a spreadsheet than to use Precision Tree. If Precision Tree is used, the resulting tree will take up several pages. It is important to review the results and isolate the major risk contributors. An example of the spreadsheet format for a similar evaluation is provided in the section on Seismic Failure of Spillway Radial (Tainter) Gates.

E-6.7   Accounting for Uncertainty

The method of accounting for uncertainty in the seismic loading is described in the sections on Probabilistic Seismic Hazard Analysis and Event Trees. Typically, the reservoir elevation exceedance probabilities are taken directly from the historical reservoir operations data, which do not account for uncertainty. Uncertainty in the failure probability and consequences are accounted for by entering the estimates as distributions (as describe above) rather than single point values. A "Monte-Carlo" simulation is then run to display the uncertainty in the estimates, as described in the section on Combining and Portraying Risks.

E-6.8   What if Pier Failure Probabilities are not independent?

As noted, the above evaluation assumes the failure probabilities for all piers are independent of each other. In reality, if a pier fails, it would make the potential failure of the remaining piers more suspect. This type of situation is addressed in the section on Seismic Failure of Spillway Radial (Tainter) Gates of this manual. An example of how the risk estimates might change if spillway gates are not independent during a seismic event is presented in that section.

E-6.9   Navigation Dams

The preceding sections have focused on dams used for flood risk management and water supply. In addition to these gated spillways, the USACE inventory of dams contains a large number of gated spillways on navigation structures. Although the potential failure mode is essentially the same regardless of the type of dam, there are some important considerations for navigation dams.

E-6.9.1   Hydrologic Loadings

While a dam designed for flood risk management will have a low pool much of the year to maintain storage capacity during flood season, a navigation dam seeks to hold a constant pool for as much time as possible to facilitate navigation on the waterway. The constant pool held by a navigation project is generally near the top of the gates, meaning the gates and piers experience their maximum load for most of the time. During high inflow events, the tailwater will tend to rise while the upper pool is held as long as possible at the constant pool. Therefore the differential loading decreases during flood events for most navigation dams.

E-6.9.2 Construction

Since a navigation dam is built on a major river, all inflow must be passed. During high flow periods this means the gates are pulled completely out of the water with a sufficient opening to prevent the gates from impeding flow. The remainder of the project (e.g. locks, navigable pass dam, etc.) however, may be designed to completely overtop during these flood events. In order to accommodate these high flood events, the gate piers must be constructed significantly taller than piers on other types of dams. In addition, it is common for navigation dams to be built with emergency or maintenance bulkheads and permanent cranes used to set them. These bulkheads are stored at the top of the piers in many cases which may add mass to the top of the piers. To illustrate, Figure E-6-8 shows a side-by-side scale comparison of a typical navigation spillway on the Ohio River, along with a typical spillway on a flood risk management dam.
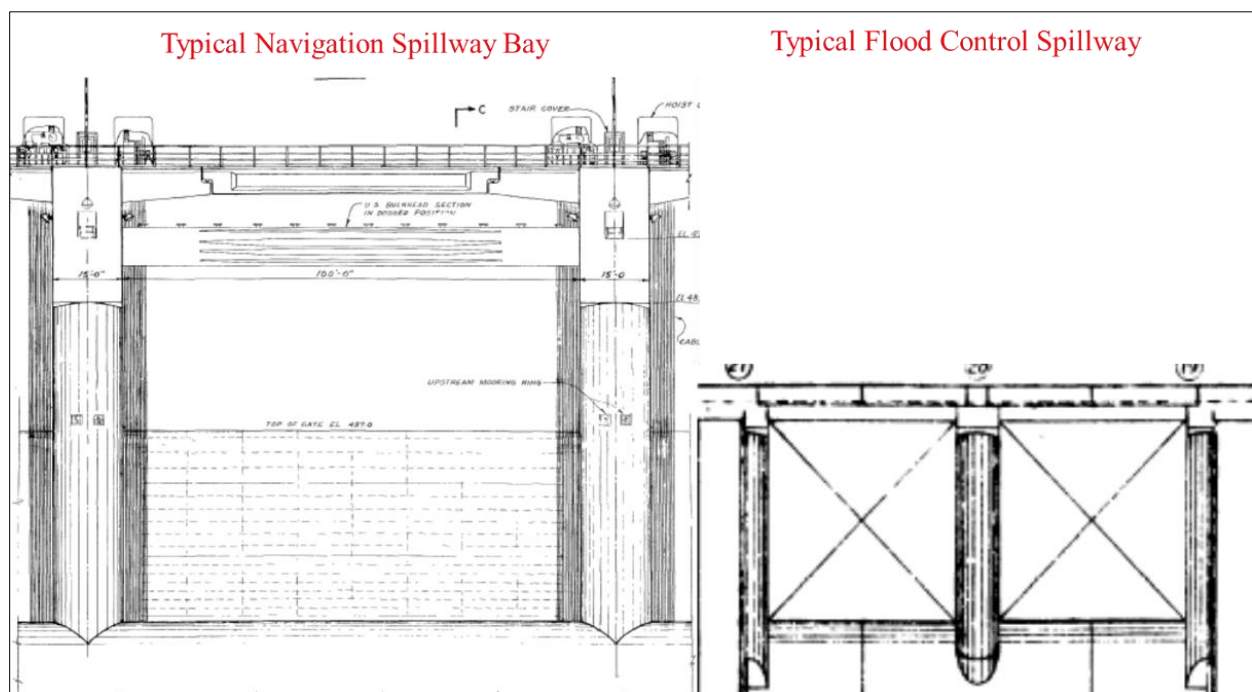
Figure E-6-8 – Comparison of Navigation and Flood Control Spillways

Although the gates are approximately the same height, the navigation gates are about twice as long, and the piers are about twice as tall. The emergency bulkhead is also shown stored at the top of the navigation pier. While the piers are generally much taller in navigation structures, the

dam itself may not be. In other words, the pier will make up a larger portion of the total dam height than for other types of dams.

E-6.9.3 Dynamic Response

Similarly, the dynamic response of the monolithic portion of a navigation dam may be limited, but the pier response will be significant. Due to the height of the piers, the natural period will tend to be longer than most other dam piers as well.

E-6.9.4 Foundations

Navigation structures located in the lower reaches of some rivers will often have significant depths of alluvial deposits. As a result, navigation structures are more often founded on deep foundations or a combination of deep and shallow foundations. The propagation of ground motions through these deep alluvial deposits and the interaction with the foundation piles must be considered in these cases.

E-6.9.5 Failure Probability

All of these factors discussed in the preceding sections may result in a relatively high failure probability for piers on navigation structures. Most navigable waterways however, do not pass through high seismic zones. Therefore, there are relatively few navigation structures throughout the USACE inventory where a seismic pier failure would have a high annual failure probability.

E-6.9.6 Economic Loss

Since navigation structures are generally able to pass very large inflow events, the downstream channel is typically very large compared to the amount of water that would be released during a breach. For this reason, most navigation structures in the USACE inventory do not have the potential for life loss. The primary consequence related to a failure would be economic loss.

In cases where only economic losses are considered, there may be other serviceability issues that result in a loss of service to the project, but that do not lead to a breach. For example, if the power lines for the locks run across the dam spillway, and movement of the pier results in a break of the lines, service to the locks will be lost for some estimated amount of time.

Another example would be if an earthquake occurs during the seasonal reduction in inflow. This condition may occur for a significant duration depending on the waterway. In these cases the gates would be in the process of closing as the inflow is tracked. If the earthquake causes small movements of the pier, this could prevent the gates from being closed. In addition to power loss, if the piers deflect enough differentially, the drive shafts for the gates could be damaged or uncoupled, preventing the gates from lowering. If the problem cannot be solved in a timely manner, the pool could be lost as the inflows continue to reduce.

These examples illustrate that when the economic impact of navigation structures are considered, without life loss consequences, a potential failure to the project may occur. This scenario considers reduced loading and subsequent damage to the piers as compared to a potential dam failure focused on a sudden breach through the spillway and related consequences.

E-6.10 Relevant Case Histories

There are no known instances of spillway pier failure during an earthquake. Despite this, the potential for this failure mode exists and is possible given a critical combination of earthquake loading, reservoir loading and the resulting stresses in the pier concrete. In lieu of failure case histories, the results of a seismic analysis of the existing spillway piers at Folsom Dam will be summarized and the design of the piers at Olmsted Dam will be discussed.

E-6.10.1   Folsom Dam

Folsom Dam was completed in 1953. The existing spillway is regulated by eight Tainter (radial) gates: five service gates that are 42 feet wide and 50-feet high and three emergency gates that are 42 feet wide and 53-feet high. The spillway gate bays are formed by 7 concrete spillway piers. The piers are 8- feet thick and vary in height from about 54 feet to 70 feet. The vertical reinforcement at the base of the piers consists of about 0.8 in^2 of steel per foot in each face. Shear reinforcing steel is not provided within the spillway piers. The trunnion anchorage for the spillway gates consists of three steel plates (2-inch by 16-inch plates, 60 feet long, which are welded together). The end anchorage for the steel plates consists of a bearing plate that is located in the mass concrete of the spillway crest concrete, below the bottom elevation of the piers.

Several structural analyses, as well as risk analyses, were performed to examine risks associated with the piers failing under seismic loading. Several potential failure modes were identified based on analysis results. The initial analyses modeled the spillway piers as part of a larger finite element model of the concrete dam. In this analysis the reservoir was modeled as fluid elements. Potential failure modes identified included: cross-canyon overstressing of the spillway piers which could lead to failure of the spillway gates, failure of concrete supporting the trunnion pins on the spillway pier due to upstream downstream loading of the gates/piers, and potentially a failure of the trunnion anchorage. The spillway gate trunnion anchorage at Folsom Dam is unique and not typical for most installations. The anchorage consists of steel plates that are not bonded to the surrounding concrete, creating a long free length for the anchorage. More refined analyses were performed, including reservoir loading in the cross-canyon evaluation of the spillway piers and modeling of the trunnion anchorage and evaluating the performance of the anchorage in a time-history analysis. The conclusions of the refined analyses were that the reservoir loading on the piers in the cross-canyon direction was significant and that the trunnion anchorage was lightly loaded, even for the largest earthquakes analyzed. The results of the risk analysis indicated that the likelihood for moment failure of the piers and shear failure of the concrete supporting the trunnion pins was high enough, that when the resulting consequences were considered, additional actions were justified. It should be noted that the analyses performed for the Folsom Dam piers and gates were high end non-linear studies that typically would only be performed when the risk estimates justified more detailed analyses.

E-6.10.2   Olmsted Dam

As noted in previous sections, there are very few navigation structures that are located in significant seismic zones. Even fewer navigation structures are new enough to have considered significant seismic loading during the original design. This means that the few navigation structures that are now considered to be located in seismically active areas were likely not designed for seismic loading. To illustrate the increased design requirements for piers in a seismic area, the new Olmsted dam design will be compared against the nearby, but older Newburgh Dam.

The Olmsted Locks and Dam are located on the lower Ohio River just upstream of the confluence with the Mississippi River. The dam includes a Tainter gate section which is a reinforced concrete structure with five piers. Each gate is 110 feet wide by 37 feet high. The Tainter gates are supported between 14-foot wide x 85-foot tall x 102-foot long piers. Due to the slow moving water in this area, the foundation is made up of very deep alluvial deposits. The structure is supported by piles which penetrate the newer, weaker, sediments. The Olmsted site is located in the outer extents of the New Madrid Seismic Zone (NMSZ), and can therefore be expected to experience large ground motions. Since a breach of Olmsted does not result in life threatening flows, the maximum design earthquake (MDE), based on USACE regulations, has an AEP of approximately 1/1000. The proximity to the NMSZ and the alluvial foundation result in a PGA at the foundation elevation of near 1g. The adopted design response spectrum is shown in Figure E-6-9.
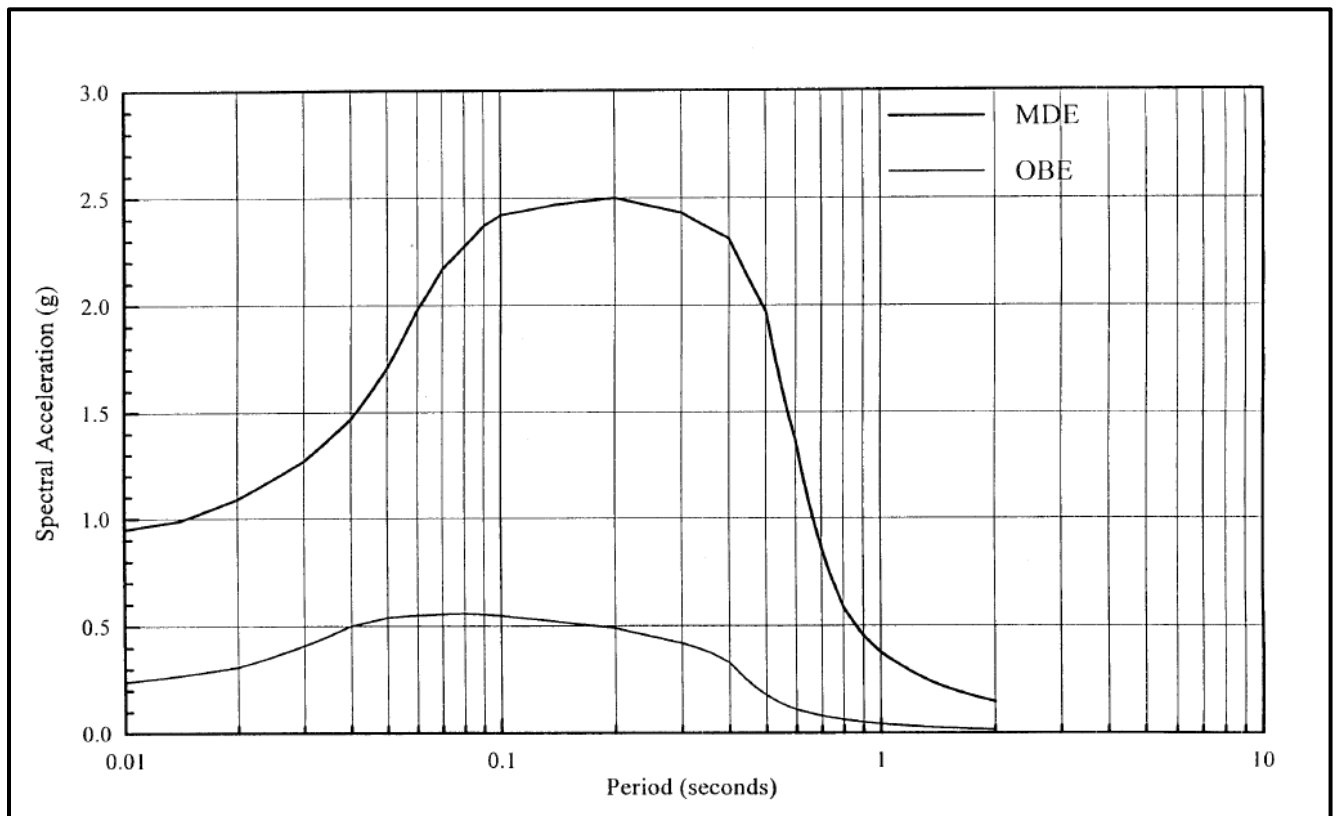

Figure E-6-9 – Olmsted Design Response Spectrum

The Olmsted Dam was built using a unique modular construction method where each component was constructed "in the dry" in a prefabrication yard. Each massive component was transported to the river using a series of enormous custom-built transport machines. The intermediate segments of the dam are massive reinforced concrete, inverted-tee shaped structures, supported on steel pipe piles. Figure E-6-10 shows the full gated spillway along with a close-up of an intermediate base shell that supports the pier. The connection of the pier to the base shell was designed to create a fixed connection between the two components, such that they act as a single unit. This results in a fundamental period of the pier of approximately 0.4 seconds in the upstream/downstream direction and 1.1 seconds in the cross-river direction.
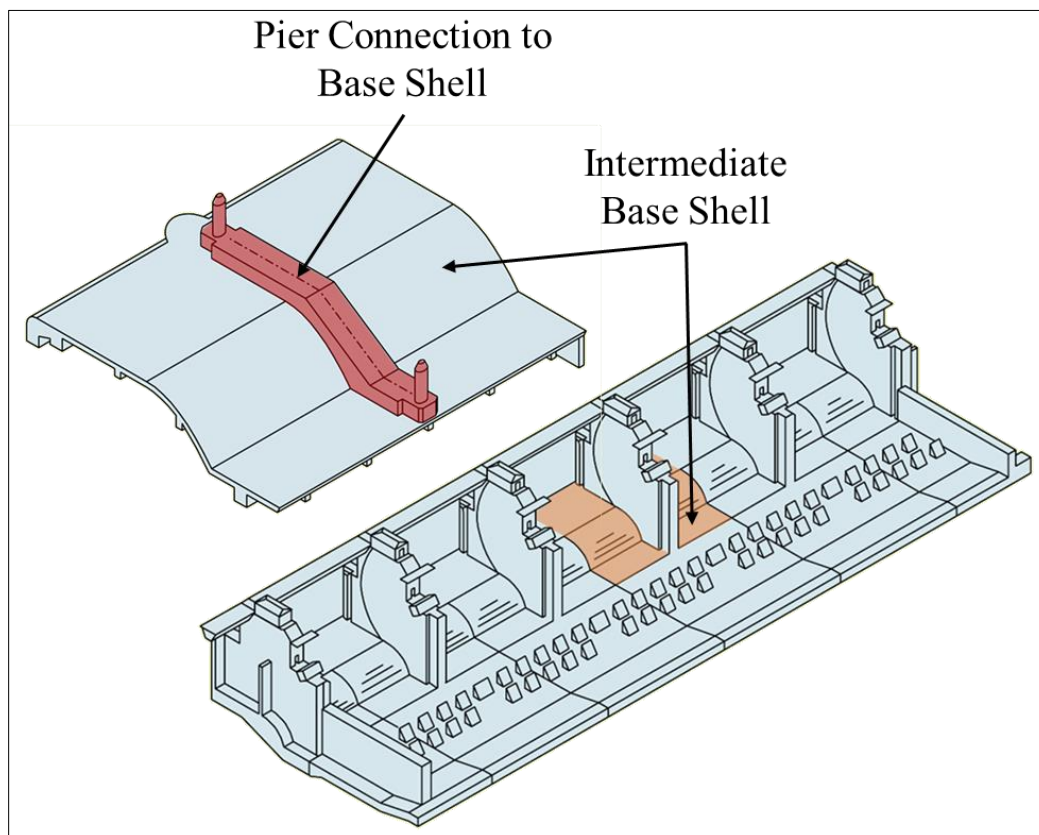

Figure E-6-10 – Modular Spillway Section

The lower Ohio River experiences high water conditions for a significant portion of the year which require the Olmsted gates to be completely raised. This high water loading, with the gates fully open, creates the controlling load case for the seismic design of the piers. The seismic

design was governed by motion in the cross-river direction. This load condition is shown in Figure E-6-11.
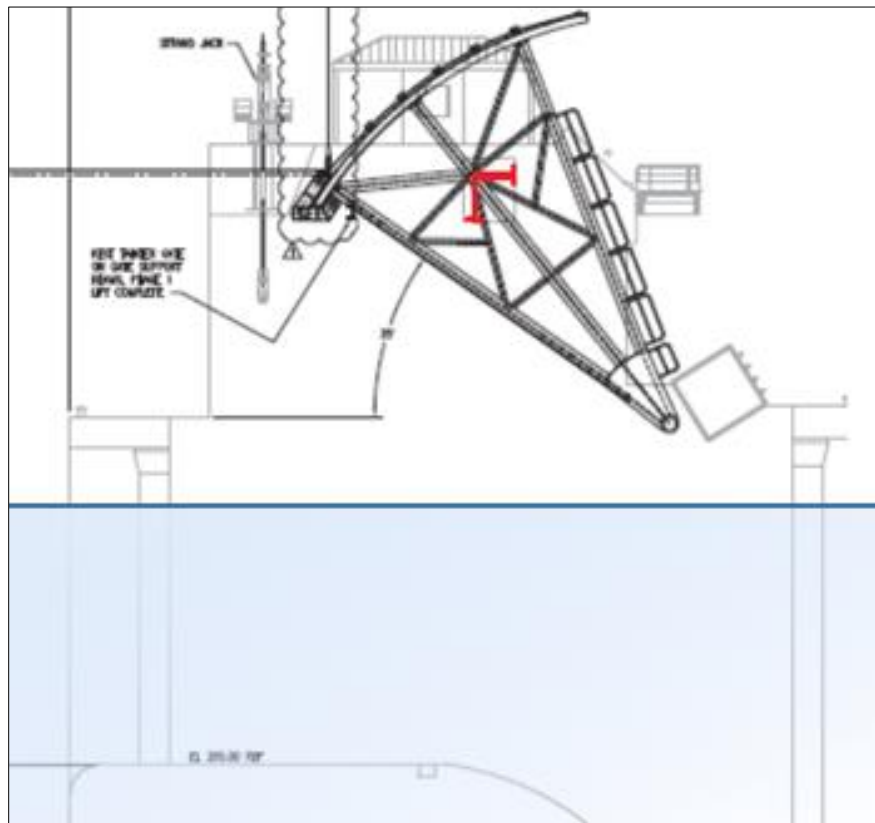

Figure E-6-11 – Controlling Water Level for Seismic Load Cases

Based on this loading condition, a modal superposition response spectrum analysis was used to design the reinforcement of the piers. Because of the thick piers used on navigation dams, in most cases where seismic loads were not considered, the reinforcement design was controlled by the minimum area of steel required by design codes. However, the minimum steel was not sufficient for the Olmsted piers due to the high seismic loading. A more typical design on the Ohio River is represented by Newburgh dam, where the minimum required steel was used. Though it was common for dams such as Newburgh to be checked for a seismic coefficient of 0.1g, this generally did not control over the minimum required steel. To illustrate the difference in reinforcement, a comparison between the primary vertical reinforcement required for the Olmsted and Newburgh piers is shown in Table E-6-4.

The shorter, slightly narrower piers at Olmsted have much heavier primary reinforcement for weak axis seismic loading than the taller piers at Newburgh. It should be noted that Newburgh Dam was designed in approximately 1960, and the Olmsted Dam was designed around 2000. Design methods and criteria changed significantly in the interim, as well as knowledge about regional seismic hazard. However, this comparison illustrates that many older dam designs may be inadequate for current seismic loading depending on the seismic hazard in the area.

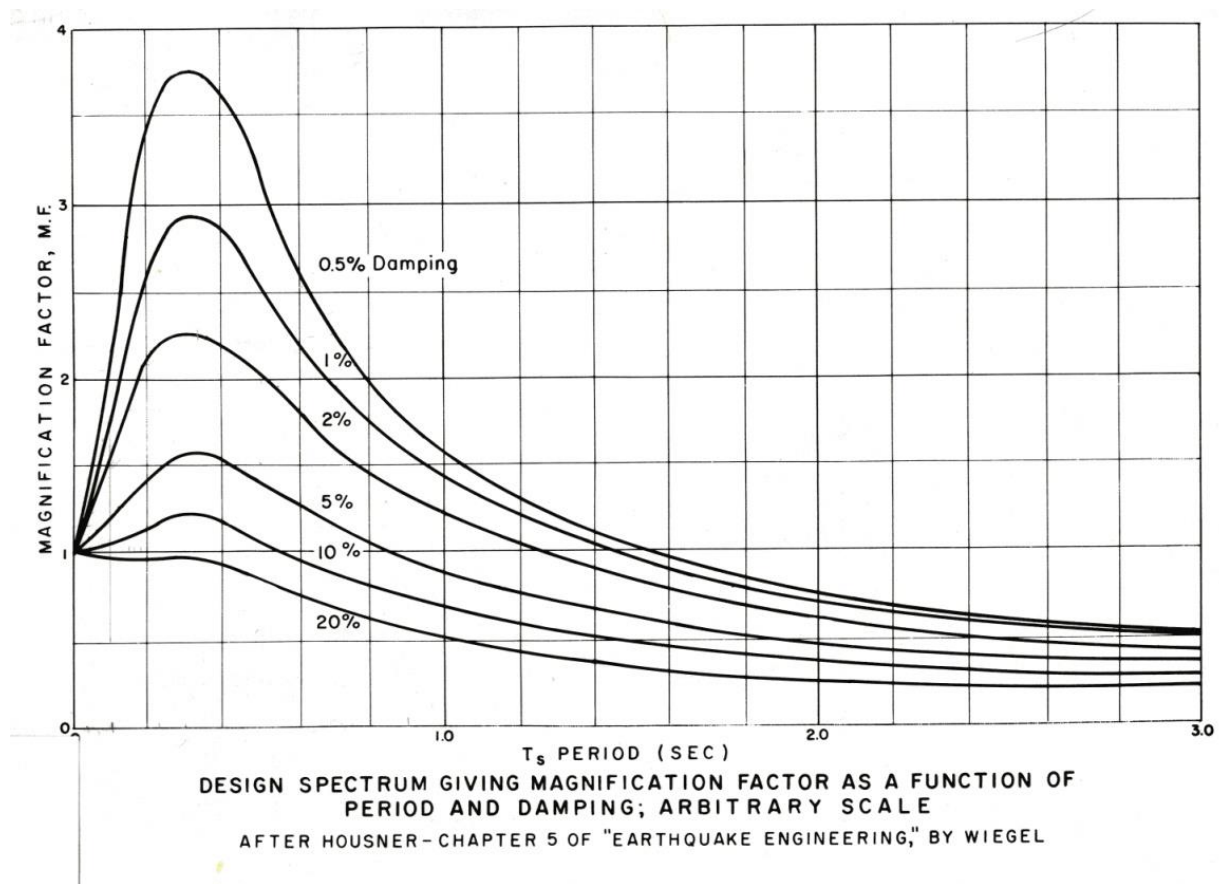**Table E-6-4 – Comparison of Seismic Pier Design (Olmsted) to Non-Seismic Design (Newburgh)**

Location, description	Newburgh	Olmsted
Pier Width	15'-0"	14'-0"
Height of pier from top of Tainter gate sill to hoist motor floor	107-ft	85-ft
Tainter Gate height	32'-0"	37'-0"
T.G. Bay side Primary vertical reinforcement, @ top of gate sill elevation	#11 @ 12"	two layers: #9@12" and #11@12"
T.G. Bay side Primary vertical reinforcement, @ trunnion pin elevation	#11 @ 12" alternating w/ #9 @ 12"	#9 @ 12", two layers

E-6.11 Exercise

Consider a spillway with concrete piers that are 5-feet thick and 40-feet high. Calculate the shear stresses at the base of the pier in the cross-canyon direction only, for the earthquakes described in Table E-6-5. Assume that the ultimate shear capacity of the spillway piers is 200 lb/in^2. Based on a comparison of the shear stress at the base of the pier to the shear capacity of the pier concrete, estimate the probability that the cross-canyon shear capacity will be exceeded for the 1000-, 5000-, 10,000-, and 50,000-year earthquake. This exercise is designed to provide input into the shear node in the middle of the event tree for this failure mode.

Table E-6-5 – Spillway Pier Analysis – Earthquake Loads

Recurrence Interval, yr	Peak Horizontal Ground Acceleration
1000	0.2g
5,000	0.4g
10,000	0.5g
50,000	0.6g

DESIGN SPECTRUM GIVING MAGNIFICATION FACTOR AS A FUNCTION OF PERIOD AND DAMPING; ARBITRARY SCALE
AFTER HOUSNER – CHAPTER 5 OF "EARTHQUAKE ENGINEERING," BY WIEGEL

Figure E-6-12 – Magnification Factor for Seismic Pier Analysis (From Reclamation, 1971)

E-6.12 References

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